EARTHQUAKE ENGINEERING AND STRUCTURAL DYNAMICS, VOL. 26, 1059–1071 (1997)

SEISMIC BEHAVIOUR OF CONFINED MASONRY WALLS

MIHA TOMAŽEVIČ* AND IZTOK KLEMENC

Slovenian National Building and Civil Engineering Institute, Dimičeva 12, 1000 Ljubljana, Slovenia

SUMMARY

The results of tests of plain and confined masonry walls with h/l ratio equal to 1·5, made at 1 : 5 scale, have been used to develop a rational method for modelling the seismic behaviour of confined masonry walls. A trilinear model of lateral resistance–displacement envelope curve has been proposed, where the resistance is calculated as a combination of the shear resistance of the plain masonry wall panel and dowel effect of the tie-columns' reinforcement. Lateral stiffness, however, is modelled as a function of the initial effective stiffness and damage, occurring to the panel at characteristic limit states. Good correlation between the predicted and experimental envelopes has been obtained in the particular case studied. The method has been also verified for the case of prototype confined masonry walls with h/l ratio equal to 1·0. Good correlation between the predicted and experimental values of lateral resistance indicates the general validity of the proposed method.

Earthquake Engng. Struct. Dyn., **26**, 1059–1071 (1997)

No. of Figures: 11. No. of Tables: 4. No. of References: 18.

KEY WORDS: confined masonry; seismic behaviour; resistance; stiffness; mathematical model

INTRODUCTION

Confined masonry, i.e. masonry with vertical tie-columns, represents one of the most widely used masonry construction systems in Europe, Asia, and Latin America. The basic feature of confined masonry structures are the vertical, reinforced-concrete or reinforced-masonry bonding elements tie-columns, which confine the walls at all corners and wall intersections, as well as along the vertical borders of door and window openings. In order to be effective, tie-columns are well connected with the bond-beams along the walls at floor levels.

It is generally believed that tie-columns prevent disintegration and improve the ductility of masonry when subjected to severe seismic loading. In a way, similar behaviour of confined masonry is expected as in the case of reinforced concrete frames with masonry infill. However, in the case of confined masonry, tie-columns do not represent the load-bearing part of a structure. According to the requirements of recent European codes, no contribution of vertical confinement to vertical and lateral resistance should be taken into account in the calculation.[1,2] The amount of reinforcement is determined arbitrarily on the basis of experience, and depends on the height and size of the building.

Seismic behaviour of confined masonry has been already subject of experimental studies. By testing a series of models of single storey masonry houses by subjecting them to dynamically imposed sinusoidal displacements with increasing amplitudes, Umek[3] concluded that vertical tie-columns significantly improve the ductility of masonry buildings, but have little effect on the lateral resistance. Similar conclusions have been obtained in Latin America by Meli[4] and Aguila *et al.*,[5] to name some typical reports. Correlation studies, where the behaviour of plain masonry wall panels of usually poor quality masonry has been compared with

* Correspondence to: Miha Tomaževič, ZAG Ljubljana, Slovenian National Building and Civil Engineering Institute, Dimičeva 12, 1000 Ljubljana, Slovenia

Contract grant sponsor: Ministry of Science and Technology of the Republic of Slovenia; Contract grant number: J2-5208-1502

CCC 0098–8847/97/101059–13$17·50

Received 20 November 1996
Revised 11 March 1997

the confined ones, have been carried out in China.[6,7] Recently, Alcocer and Meli[8] studied energy dissipation capacity and ductility of masonry wall assemblages, confined and reinforced in different ways, whereas Aguilar *et al.*,[9] Iiba *et al.*[10] and Yoshimura *et al.*[11] studied the influence of horizontal reinforcement on the seismic behaviour. Moroni *et al.*[12,13] collected a data base of 50 tests of confined masonry walls in Latin America and proposed a simple model, based on the statistical analysis of test results, to predict their inelastic behaviour. They also conducted a parametric study and investigated the requirements regarding displacement capacity and storey drift.

To obtain the basic data needed for the evaluation of results of shaking-table tests of models of confined masonry buildings, recently carried out at Slovenian National Building and Civil Engineering Institute (ZAG),[14] 3 confined masonry model walls, built at 1:5 scale, have been tested within the experimental part of the research project. In addition, 3 referential plain masonry walls have been tested under similar loading conditions to obtain the basic mechanical properties of the model masonry. Although the tested walls have been relatively small, the expected mechanism of seismic behaviour has been adequately reproduced. Taking advantage of experimental results, the observed behaviour and measured data have been used to develop a rational method for modelling the seismic behaviour of confined masonry wall panels.

TESTS AND OBSERVED MECHANISM

In the specific case used for the verification of the proposed mathematical model, 380/240/38 mm model walls, either confined at the vertical borders of the wall with 20/38 mm r.c. tie-columns, reinforced with two Φ 3·2 mm diameter reinforcing bars (specimens AH), or without tie-columns (specimens BH) have been tested (Figure 1). Although the walls cannot be classified as squat, their height to length ratio $h/l = 1{\cdot}58$ indicated that predominant shear behaviour can be expected when subjected to lateral loading.

Model masonry blocks with dimensions 78/38/58 mm, corresponding to 39/19/29 cm prototype blocks, have been made of a special mix, composed of crushed brick aggregate, hydrated lime and Portland cement in the proportion of 1·06 : 3·45 : 81. Mortar composed of Portland cement, hydrated lime and river-bed sand in the proportion of 0·4 : 1 : 11 has been used for the construction of the walls. Micro-concrete composed of Portland cement PC-45 and river-bed sand in the proportion of 1 : 5, has been used for casting the vertical confining elements. Compression strength of blocks f_b, mortar f_m and concrete f_c was 1·09, 0·45 and

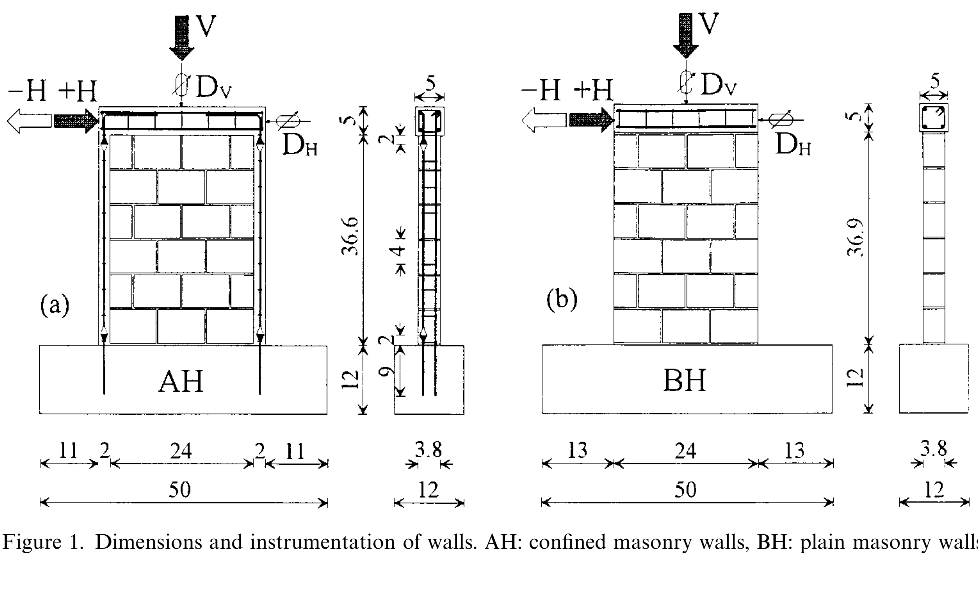

Figure 1. Dimensions and instrumentation of walls. AH: confined masonry walls, BH: plain masonry walls

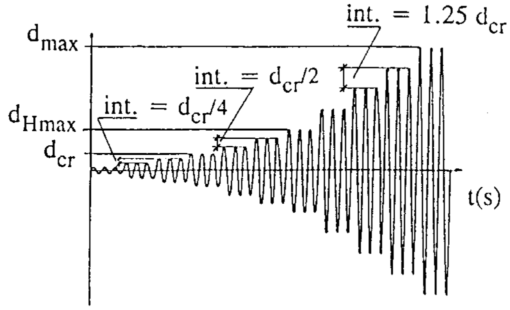

Figure 2. Imposed displacements' pattern

10·8 MPa, respectively. Commercially available fully annealed wire, with yield and tensile strength equal to $f_y = 266$ MPa and $f_r = 344$ MPa, respectively, has been used to reinforce the tie-columns. Compression f and tensile strength of masonry f_t, obtained by testing unreinforced masonry walls at compression and shear, was $f = 1{\cdot}27$ MPa and $f_t = 0{\cdot}12$ MPa. Modulus of elasticity of masonry, determined at $\frac{1}{3}$ of compression strength, was $E = 942$ MPa. Shear modulus was $G = 185$ MPa.

In order to study the seismic behaviour, 3 specimens of each, confined and plain masonry walls, designated AH and BH, respectively, have been tested as symmetrically fixed in a test set-up, which made possible the application of constant vertical load and programmed pattern of cyclically acting horizontal displacements (Figure 2), while keeping the lower and upper boundary of the specimen parallel. Average compression stress $\sigma_0 = 0{\cdot}28$ MPa, approximately 22% of the masonry's compression strength, has been induced by a constant gravity load. Instrumentation of specimens AH and BH is presented in Figure 1.

Shear failure occurred in all cases. Whereas ductile behaviour has been observed in the case of confined specimens AH, the behaviour of plain masonry walls BH was brittle. Uniformly distributed diagonally oriented cracks formed in the case of confined specimens AH. Subjected to increased lateral displacements, however, the cracks along the diagonals passed into the tie-columns. Masonry units in the middle part of the wall and concrete of tie-columns at the corners started crushing before the final collapse of the specimens, which was mainly due to the buckling of one or both vertical confining elements. One single diagonal crack developed in the case of walls BH, leading to a sudden collapse along a clearly formed diagonal failure plane at much smaller displacement amplitudes than in the case of confined masonry specimens. Typical outlook of specimens AH and BH just before, or during collapse, is shown in Figure 3.

The observed degree of damage has been used as one of the reference parameters in the modelling of seismic behaviour of the walls. In the case studied, four distinct degrees of damage have been distinguished and corresponding damage indexes defined:

(1) $I_d = 0{\cdot}25$: The formation of first diagonally oriented cracks in the middle part of the wall, passing through horizontal and vertical mortar joints. Elastic (crack) limit.
(2) $I_d = 0{\cdot}50$: Increased number of cracks, oriented diagonally in both diagonal directions. Generally, the cracks are passing through horizontal and vertical mortar joins. Usually, this type of crack pattern is observed at the attained maximum lateral resistance of the panel.
(3) $I_d = 0{\cdot}75$: Heavy damage. Increased number of wide diagonally oriented cracks, passing also through masonry units. Crushing of individual masonry units and shearing of concrete of tie-columns at the upper section of the wall.
(4) $I_d = 1{\cdot}00$: Increased width of cracks, passing through masonry units, with crushing of units along both wall diagonals. Crushing of concrete at the upper section of the tie-columns. Rupture of reinforcing bars, or buckling and collapse of tie-columns. Final collapse of the specimen.

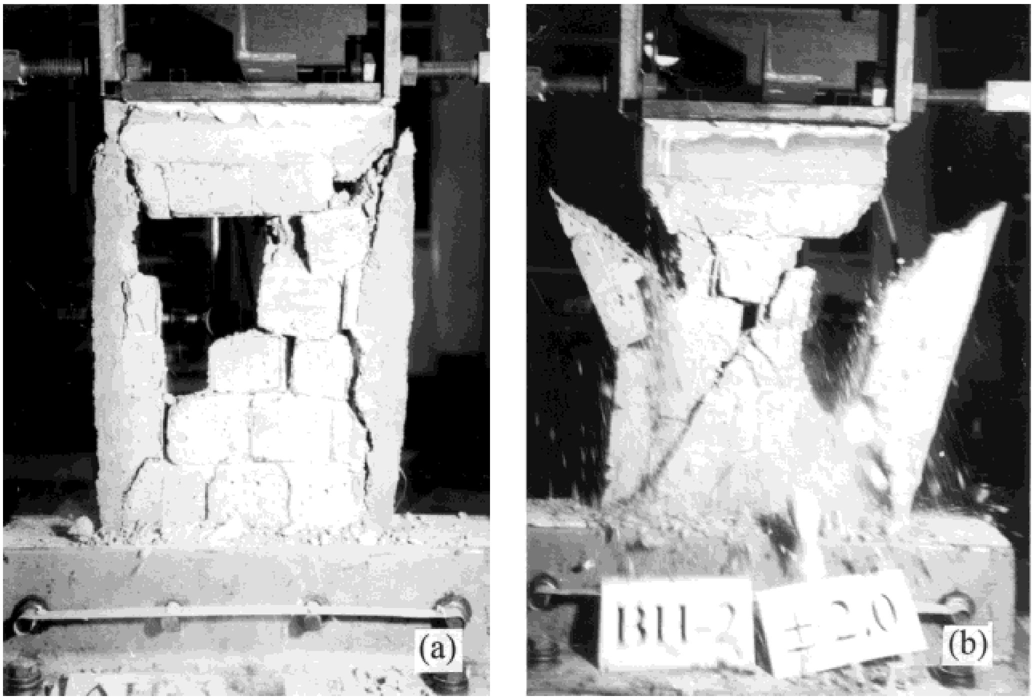

Figure 3. Typical walls after lateral resistance test. Confined (a) and plain masonry wall (b)

RESISTANCE ENVELOPE

Typical hysteresis loops, showing the relationship between the measured lateral load and displacements for confined and plain masonry walls in seismic situation, are shown in Figure 4. The resistance envelopes, i.e. the envelopes of hysteresis loops of all tested walls are compared in Figure 5. It can be seen that, by confining the wall with r.c. tie-columns, lateral resistance and deformation capacity of a plain masonry wall is significantly improved. In the particular case studied, the resistance has been improved by more than 1·5-times and the deformation capacity by almost 5-times. The analysis has shown, that, by preventing the

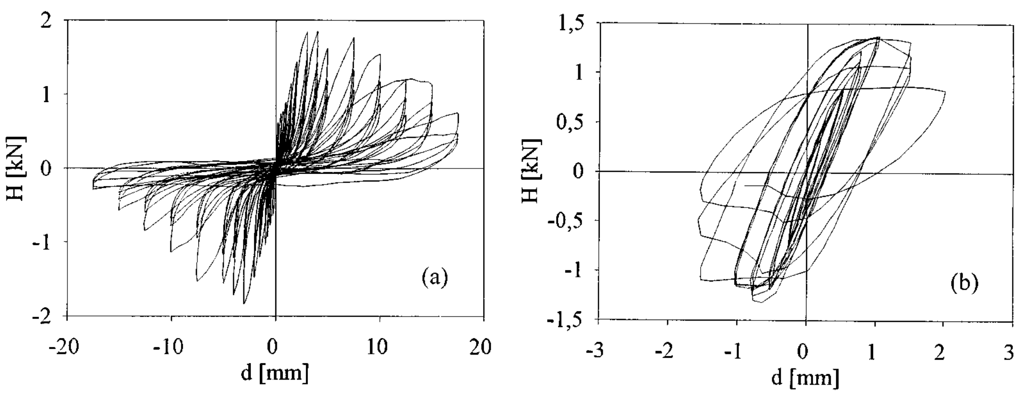

Figure 4. Typical lateral load–displacement hysteresis loops. Confined masonry wall–specimen AH-3 (a) and plain masonry wall–specimen BH-3 (b)

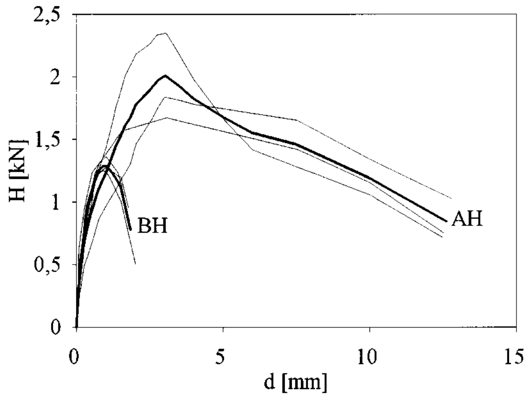

Figure 5. Comparison of hysteresis envelopes obtained by lateral resistance tests of confined (specimens AH) and plain masonry walls (specimens BH)

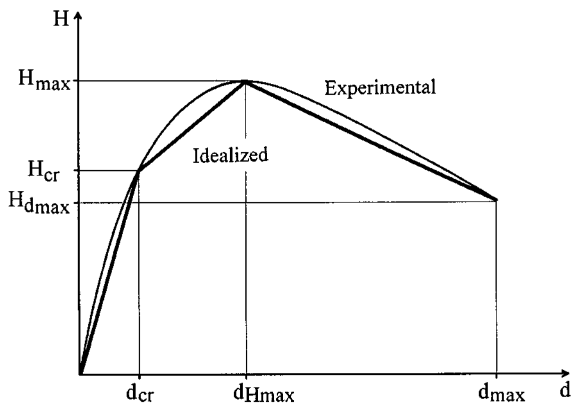

Figure 6. Idealization of experimentally obtained hysteresis envelope

disintegration of the masonry, tie-columns increased the energy dissipation capacity of the plain masonry wall by 6–7 times.

Lateral resistance

To model the seismic behaviour of a confined masonry wall, experimentally obtained resistance envelope, representing the relationship between the lateral load and displacement during the seismic response of the structure, is idealized with a trilinear relationship, shown in Figure 6. Three characteristic points (limit states) are defined on the experimental curve, determined by three pairs of parameters (Table I):

(a) *elastic* (*crack*) *limit*: determined by lateral load (H_{cr}) and displacement (d_{cr}) at the formation of the first significant crack in the wall, which changes the initial stiffness,
(b) *maximum resistance*: determined by lateral load (H_{max}) and displacement ($d_{H\max}$) at the attained maximum resistance of the wall, and
(c) *ultimate state*: determined by lateral resistance ($H_{d\max}$) at maximum attained displacement (d_{max}) just before collapse.

Table I. Parameters of experimentally obtained hysteresis envelopes

Wall's designation	H_{cr} (kN)	d_{cr} (mm)	H_{max} (kN)	d_{Hmax} (mm)	H_{dmax} (kN)	d_{max} (mm)	μ_u
AH-1	1·14	0·63	2·29	3·03	0·72	12·52	3·56
AH-3	0·64	0·27	1·84	3·02	0·52	17·53	7·59
average AH	0·89	0·45	2·07	3·03	0·62	15·03	5·58
BH-1	0·96	0·35	1·28	0·93	1·01	1·53	2·78
BH-2	0·76	0·26	1·36	1·02	0·88	1·77	3·12
BH-3	0·82	0·34	1·36	0·88	0·96	1·77	3·03
average BH	0·84	0·32	1·33	0·94	0·95	1·69	2·98

The values, given in Table I, are average values obtained at loading in positive and negative direction.

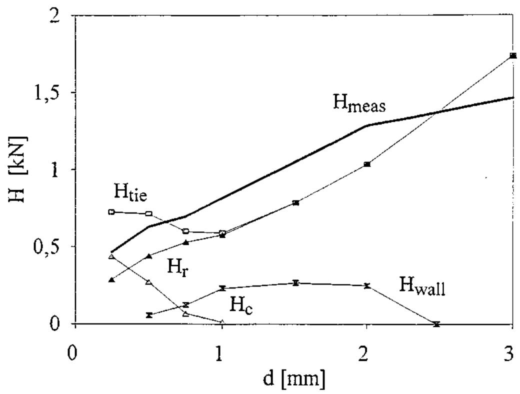

Figure 7. Typical distribution of lateral load between tie-columns and masonry wall panel

Correspondingly, the expressions for resistance and displacement at characteristic points of the idealized envelope should be developed.

As can be seen by analysing the mechanism of action of vertical confinement, strains in reinforcing bars changed alternatively from tension to compression, indicating monolithic behaviour of the confined masonry wall at small displacement amplitudes. By knowing the forces induced in the confinement, bending moments at the bottom and top of the wall have been evaluated, and the contribution of lateral loading, carried by the tie-columns, estimated. Typical results of such analysis are presented in Figure 7, where the calculated resistance of vertical tie-columns H_{tie}, assuming the monolithic behaviour of the confined panel is correlated with the measured lateral load H_{meas}–displacement d curve. In the calculation of the resistance of tie-columns, the contribution of concrete H_c has been added to the contribution of reinforcement H_r. The assumed contribution of masonry wall is obtained as a difference $H_{wall} = H_{meas} – H_{tie}$.

As can be seen in Figure 7, the assumption of monolithic behaviour of a confined masonry wall panel is acceptable in the linear range. In order to be consistent with the main structural material, the equation proposed for the calculation of the shear resistance of plain masonry walls,[15] has been used to assess the resistance of the panel:

$$H_{u,s} = \frac{f_t A_w}{b} \sqrt{\frac{\sigma_0}{b} + 1} \tag{1}$$

where $H_{u,s}$ is the shear resistance of the wall, A_w the area of the horizontal cross-section of the wall, σ_0 the average compression stress in the horizontal cross-section of the wall, and b the shear stress distribution coefficient.

According to equation (1), shear resistance of a masonry wall depends on the level of compression stresses due to gravity loads. In a confined panel, however, interaction forces develop between the confining elements and masonry at the contact zones, which induce additional compression stresses:

$$\sigma_0 = \sigma_{0,v} + \sigma_{0,i} \tag{2}$$

where

(i) $\sigma_{0,v} = V_w/A_w$ is the compression stress in the masonry wall due to part of imposed vertical load V, which is carried by the masonry wall panel. In the case of tested walls, where the ratio of concrete to masonry cross-section area was relatively high, and the bond-beam on the top of wall was rigid, $V_w = 0{\cdot}4V$. In normal cases, $V_w = V$.
(ii) $\sigma_{0,i}$ the additional compression stress in the masonry wall due to interaction between the masonry and confinement.

The analogy with masonry infilled r.c. frames has been used in order to estimate the level of interaction forces.[16,17] The shape and distribution of interaction forces between the r.c. tie-columns and masonry panel is schematically presented in Figure 8. By assuming that the lateral load is carried by the wall, and taking into account the equilibrium of moments of interaction forces, the following relationship between the lateral load H and resultant of vertical interaction forces V_i can be obtained.

$$V_i = \frac{H s_w}{\alpha} \tag{3}$$

where V_i is the resultant of vertical interaction forces, causing additional compressive stresses in the horizontal cross-section of the wall, s_w the geometry aspect ratio (shape factor $s_w = h/l$), α the parameter which depends on the assumed shape and distribution of interaction forces, induced by lateral loading H (see Figure 8). It can be expressed as

$$\alpha = \frac{(x_{Vz} - x_{Vs})h}{y_H I} \tag{4}$$

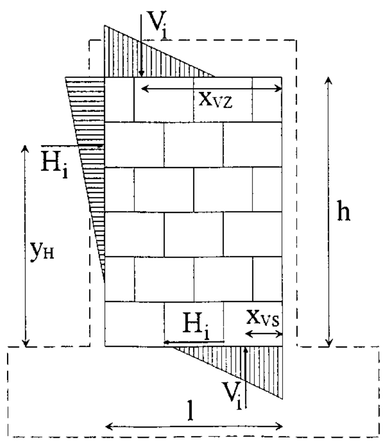

Figure 8. Shape and distribution of interaction forces between r.c. tie-columns and masonry panel

On the basis of damage observation, measurements and geometry of the tested walls, the value of parameter $\alpha = 5/4$ has been proposed to be taken into account in further calculations. Taking this into consideration, vertical compressive stresses in the masonry wall due to interaction between the tie-column and masonry can be calculated by

$$\sigma_{0,\mathrm{i}} = \frac{V_\mathrm{i}}{A_\mathrm{w}} = H \frac{s_\mathrm{w}}{\alpha A_\mathrm{w}} \tag{5}$$

Equation (2) is used to calculate the stresses in the masonry wall panel due to imposed vertical load and interaction forces between the confinement and masonry wall panel due to lateral load. If equation (2) is introduced into equation (1), the following final form of equation, determining the contribution of a masonry wall, confined with tie-columns, to the lateral resistance of confined masonry wall system, is obtained:

$$H_{\mathrm{u,s}} = \frac{f_\mathrm{t} A_\mathrm{w}}{C_\mathrm{i} b}\left[1 + \sqrt{C_\mathrm{i}^2\left(1 + \frac{V_\mathrm{w}}{f_\mathrm{t} A_\mathrm{w}}\right) + 1}\right] \tag{6}$$

where C_i is the interaction coefficient,[17] which takes account for the distribution of interaction forces, as well as the distribution of shear stresses along the horizontal cross-section of the masonry wall panel:

$$C_\mathrm{i} = 2\alpha b \frac{1}{h} \tag{7}$$

Since the contribution of tie-columns' reinforcement in the small deformation range is small, it is not taken into consideration. By definition, tensile strength of masonry is represented by maximum referential principal tensile stress, developed in the referential elastic wall at the attained maximum resistance. Therefore, if equation (6) is used to assess the resistance of the confined masonry wall at the attained elastic limit (at the formation of first cracks), the calculated shear resistance of the panel should be reduced by reduction factor C_cr, indicating the experimentally observed value of crack limit to maximum wall resistance ratio ($C_\mathrm{cr} = H_\mathrm{cr}/H_{\mathrm{u,s}} = 0{\cdot}7 - 0{\cdot}8$):

$$H_\mathrm{cr} = C_\mathrm{cr} H_{\mathrm{u,s}}. \tag{8}$$

However, as soon as the damage progresses, the assumption of monolithic behaviour of confined masonry wall panel is no longer valid. At larger amplitudes, tension in tie-columns' reinforcement prevailed at both directions of imposed lateral displacements. Namely, once cracked, the masonry panel pushed the tie-columns sidewards and induced tension in the reinforcing bars. Vice versa, r.c. confining elements prevented the disintegration of masonry and induced additional compression stresses in both, vertical and horizontal, directions. However, the analysis has shown that the dowel action of tie-columns' reinforcement improves the resistance of the confined masonry panel at maximum resistance. The contribution of tie-columns' reinforcement due to dowel action is calculated by[18]

$$H_{d,\mathrm{r}} = \sum_1^n 0{\cdot}8059\, d_\mathrm{r}^2 \sqrt{f_\mathrm{c} f_\mathrm{y}} \tag{9}$$

where n is the number of reinforcing bars, d_r the diameter of reinforcing bar, f_c the compressive strength of concrete and f_y the yield stress of reinforcing steel.

Consequently, maximum resistance of a confined masonry wall is obtained as a sum of contributions of the masonry wall panel (equation (6)) and r.c. confining elements due to dowel action of reinforcing steel (equation (9)):

$$H_\mathrm{max} = H_{\mathrm{u,s}} + H_{d,\mathrm{r}} \tag{10}$$

Not enough data have been obtained to physically model the strength degradation after the attained maximum resistance and to calculate the remaining resistance of the wall at the ultimate limit just before collapse. For practical reasons, the value of shear resistance capacity, calculated by equation (10), is reduced by an experimentally obtained strength reduction factor C_{sr}, representing the ratio between the remaining resistance of the wall at ultimate limit and maximum resistance:

$$H_{d\max} = C_{sr} H_{\max} \tag{11}$$

In the particular case studied, the value of $C_{sr} = 0{\cdot}4$ has been evaluated from the test results. This value is relatively low and is a consequence of relative slender walls that have been tested in this particular study. In the case of squat walls, strength degradation of more than 30–40 per cent of maximum resistance, i.e. $C_{sr} = 0{\cdot}7$–$0{\cdot}6$ has been rarely observed. Consequently, in most practical cases, ultimate strength degradation below 70 per cent of maximum resistance capacity, i.e. C_{sr} less than 0·7, is not acceptable.

Stiffness

Masonry is an inelastic structural material, which does not behave elastically even in the range of small deformations. However, for practical reasons, effective stiffness of the wall K_e, defining the slope of the first branch of the idealized resistance envelope, is determined as the ratio between the lateral load and displacement at the formation of the first significant crack in the wall:

$$K_e = H_{cr}/d_{cr}. \tag{12}$$

After cracking, stiffness of the panel is defined as the secant stiffness $K = H/d$, i.e. as the ratio between the lateral resistance of the wall and corresponding displacement. The values of secant stiffness, calculated at characteristic points of lateral load–displacement hysteresis envelopes (d_{cr}, $d_{H\max}$ in $d_{\max}$), are given in Table II for both types of tested walls. In order to estimate the degree of stiffness degradation, the calculated values are also expressed in terms of the effective stiffness of the wall K_e. Stiffness degradation in dependence on the imposed lateral displacements, obtained in the case of confined and plain masonry walls, is plotted in Figure 9.

Since the degree of stiffness degradation is obviously depending on the observed degree of damage, correlation between the damage index I_d and secant stiffness of the wall K, has been analysed. In the analysis, secant stiffness of the wall has been expressed with the effective stiffness of the wall K_e, which can be easily calculated by the well known methods of the theory of structures. This way, a general validity is given to the proposed equation.

Table II. Stiffness degradation at characteristic points of hysteresis envelopes

Specimen designation	K_e (kN/mm)	$K_{H\max}$ (kN/mm)	$K_{H\max}/K_e$	$K_{d\max}$ (kN/mm)	$K_{d\max}/K_e$
AH-1	1·82	0·76	0·42	0·06	0·03
AH-2$^{(-)}$	1·70	0·54	0·32	0·02	0·01
AH-3	1·46	0·61	0·42	0·03	0·02
Average AH	1·66	0·64	0·38	0·04	0·02
BH-1	2·78	1·35	0·48	0·25	0·09
BH-2	2·91	1·33	0·46	0·50	0·17
BH-3	2·40	1·45	0·60	0·54	0·22
Average BH	2·70	1·38	0·51	0·43	0·16

Note: Values in Table III are average values, obtained at loading in positive and negative direction. In the case of specimen AH-2, only negative branch of loading has been taken into account

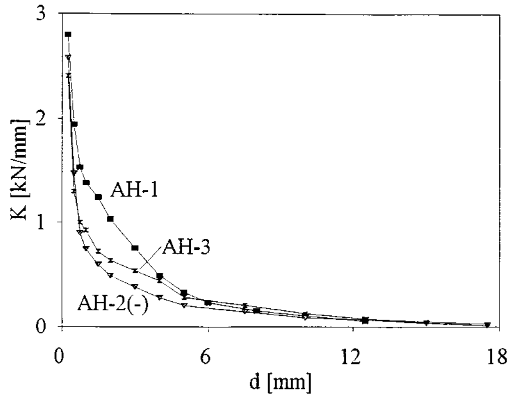

Figure 9. Stiffness degradation of confined masonry walls in dependence on imposed lateral displacements

Table III. Calculated parameters of hysteresis envelope

Limit state	I_d	H (kN)	K (kN/mm)	d (mm)
Elastic limit	0·25	1·11	3·45	0·32
Max. resistance	0·50	2·39	1·50	1·60
Ultimate state	1·00	0·96	0·07	14·08

The following boundary conditions have been considered in the analysis:

(a) effective stiffness K_e of the wall is defined as the secant stiffness at the occurrence of first significant cracks, i.e. at the damage to the wall, corresponding to $I_d = 0{\cdot}25$;
(b) ultimate stiffness of the wall just before collapse, at the damage corresponding to $I_d = 1{\cdot}00$, amounts to 2 per cent of effective stiffness K_e (see Table III).

Taking this into consideration, a two-parameter equation of the form:

$$K = K_e - \sqrt{aI_d - b} \tag{13}$$

where a and b are the stiffness degradation parameters, has been proposed. Taking into account the assumed boundary conditions and experimental results, the values of stiffness degradation parameters $a = 1{\cdot}281K_e^2$ and $b = 0{\cdot}320K_e^2$ have been obtained, which gives equation (13) the final form:

$$K = K_e(1 - \sqrt{1{\cdot}281I_d - 0{\cdot}320}) \tag{13a}$$

The comparison between the experimentally observed and calculated relationship between the damage to the walls and stiffness degradation is shown in Figure 10. As can be seen, good correlation between the experimental and predicted relationship has been obtained.

The stiffness of the wall depends on dimensions, mechanical properties of masonry materials, and boundary conditions. Effective stiffness of the confined masonry wall panel is calculated by simple equation, based on the theory of elasticity, which takes account for flexural and shear deformations of the wall:

$$K_e = \left(\frac{h^3}{\beta EI_w} + \frac{\kappa h}{GA_w}\right)^{-1} \tag{14}$$

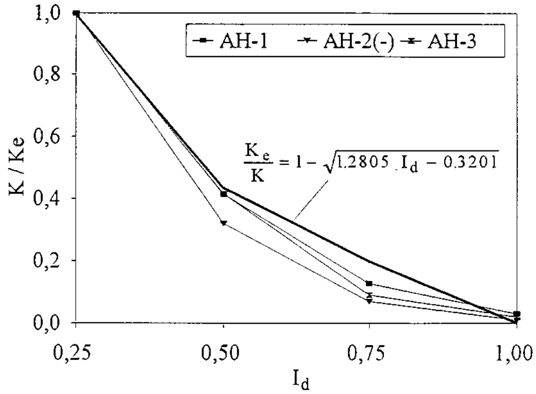

Figure 10. Correlation between experimental and predicted stiffness degradation of confined masonry walls in dependence on damage index

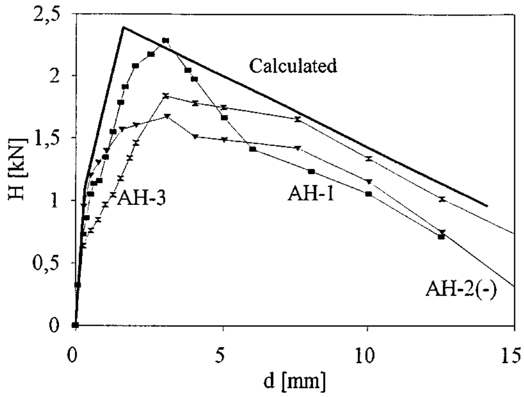

Figure 11. Correlation between experimental and calculated lateral resistance envelopes

where I_w is the moment of inertia of the horizontal cross-section of the wall, β the coefficient depending on boundary restraints ($\beta = 12$ in the case of fixed-ended and $\beta = 3$ in the case of cantilever wall) and κ the shear coefficient of the section.

Since the dimensions of r.c. confining elements are relatively small, their contribution to lateral stiffness is not taken into account explicitly. However, in the calculation of the moment of inertia of the section, gross sectional area of the confined panel, including the dimensions of the tie-columns, with attributed masonry mechanical characteristics, should be considered. Equation (13a) is taken into account to estimate the values of stiffness of the wall at maximum resistance (damage index $I_d = 0{\cdot}50$) and ultimate state (damage index $I_d = 1{\cdot}00$).

By using equations (8), (10), and (11), geometry of confined masonry wall, indicated in Figure 1, as well as the actual values of mechanical properties of masonry materials and reinforcing steel, lateral resistance of the tested confined masonry walls at characteristic limit states has been calculated. Equations (14) and (13a) have been used to evaluate the stiffness. The calculated values are given in Table III, where the displacements of the wall, corresponding to characteristic limit states, are also evaluated. The calculated hysteresis envelope is compared with experimentally obtained curves in Figure 11. As can be seen, good correlation between the

Table IV. Correlation between experimentally obtained and calculated values of parameters of resistance envelope[9,12]

Wall designation	h/l ratio	σ_0/f ratio	H_{cr} (kN)			H_{max} (kN)		
			Exp.	Calc.	Exp./Calc.	Exp.	Calc.	Exp./Calc.
Wall 18[12]	1·0	0·093	55·9	95·0	0·58	119·5	92·7	1·29
Wall 19[12]	1·0	0·046	139·0	164·1	0·85	190·0	217·9	0·87
Wall 20[12]	1·0	0·059	164·5	162·1	1·01	186·0	215·5	0·86
Wall 21[12]	1·0	0·046	132·5	164·1	0·81	167·6	217·9	0·77
Wall 27[12]	1·0	0	151·9	190·6	0·80	204·0	251·0	0·81
Wall 28[12]	1·03	0	146·0	154·6	0·94	189·0	206·1	0·92
Wall 29[12]	1·03	0·025	173·0	167·1	1·04	193·8	221·7	0·87
Wall 30[12]	1·03	0·08	182·0	188·8	0·96	236·5	248·8	0·95
Wall 31[12]	1·03	0·13	213·3	207·4	1·03	235·2	272·0	0·86
Wall MO[9]	1·01	0·14	70·8	70·9	1·00	114·0	115·0	0·99

experimental and idealized envelopes has been obtained in this particular case, indicating the validity of the proposed model.

To estimate the model's general validity, experimental results obtained by testing prototype size confined masonry wall panels, published in the literature,[9,12] are correlated with calculated values in Table IV. Since not all necessary data regarding the masonry material characteristics and reinforcing details can be found in the papers, only a limited number of parameters has been verified. As can be seen, although the proposed model, developed on the basis of tests of reduced size walls with height to length ratio $h/l = 1{\cdot}5$, has been applied to prototype walls with ratio $h/l = 1{\cdot}0$, good correlation between the measured and calculated resistance values has been obtained. The analysis of correlation indicates that the calculated values of resistance at cracking and maximum resistance are overestimated by an average of 10 and 8 per cent, respectively. However, since the reported testing details do not permit the harmonization of the values of material characteristics with the values used in the proposed model, the conclusion should be considered as indicative.

CONCLUSIONS

The mechanism of seismic behaviour of confined masonry wall panels with h/l ratio equal to 1·5 has been experimentally studied by testing a number of confined masonry walls under seismic loading conditions. Although built of model materials at a reduced, 1:5 scale, the basic relationships between the mechanical characteristics of masonry and concrete of the model walls remained the same as in the case of the usual prototype construction. This made possible the evaluation of test results and formulation of mathematical model for seismic resistance analysis. In order to directly estimate the effect of confining the plain masonry wall with vertical tie-columns on the seismic behaviour, the same number of plain masonry walls with the same geometrical and material characteristics have been tested as referential specimens by subjecting them to similar loading conditions.

On the basis of the observed behaviour and analysis of test results, a model for the prediction of lateral resistance–displacement envelope curve, idealized by a trilinear relationship, defined with elastic limit, maximum resistance, and ultimate state of the wall, has been proposed. Monolithic behaviour and predominant shear mechanism are the basis of the model at small amplitudes of deformation. After cracking of masonry at maximum resistance, however, the contribution of tie-columns' reinforcement due to dowel mechanism is taken into account. Monolithic behaviour and actual boundary restraints are assumed in the case of determination of effective stiffness of the confined masonry wall panel. Stiffness degradation has been

modelled as a function of effective stiffness of the panel and damage index at characteristic limit states. Good correlation between predicted and experimental envelopes has been obtained, which indicates the validity of the proposed mathematical model.

ACKNOWLEDGEMENTS

The research discussed in this contribution has been financed by the Ministry of Science and Technology of the Republic of Slovenia (project J2-5208-1502). As regards the experimental part of the project, the contribution of the author's colleague Mr. Ljubo Petković is gratefully acknowledged.

REFERENCES

1. Eurocode 6. *Design of Masonry Structures. Part 1-1: General rules for buildings.* Rules for reinforced and unreinforced masonry. ENV 1996-1-1: 1995, 1995.
2. Eurocode 8. *Design Provisions for Earthquake Resistance of Structures. Part 1-3: General Rules—Specific Rules for Various Materials and Elements.* ENV 1998-1-3: 1995, 1995.
3. A. Umek, 'Comparison between unreinforced, confined and horizontally reinforced masonry walls', *Civil Engng. J.* (Ljubljana) **20**, 241–248 (1971) (in Slovene).
4. R. Meli, 'Behaviour of masonry walls under lateral loads', *Proc. 5th World Conf. Earthquake Engng.*, Rome, paper 101a, 1973.
5. V. Aguila, F. Delfin and M. Astroza, 'Estudio experimental de soluciones de reparacion y refuerzo para elementos de albanileria', *Pub. SES I 1/88 (221)*, Universidad de Chile, Santiago, 1988.
6. Z. Bolong, W. Mingshun and Z. Deyuan, 'Shaking table study of a five-story unreinforced block masonry model building strengthened with reinforced concrete columns and tie bars', *Proc., US-PRC Joint Workshop on Seismic Resistance of Masonry Struct.*, Harbin, IV-11; 1–11, 1988.
7. Y. Wenzhong and J. Zhaohong, 'Functions of ties concrete columns in brick walls', *Proc. 9th World Conf. Earthquake Engng.*, Tokyo-Kyoto, Vol. 6, 1988, pp. 139–144.
8. S. Alcocer and R. Meli, 'Test program on the seismic behaviour of confined masonry walls', *The Masonry Soc. J.* (Boulder) **13** (2), 68–76 (1995).
9. G. Aguilar, R. Meli, R. Diaz and R. Vasquez-del-Mercado, 'Influence of horizontal reinforcement on the behaviour of confined masonry walls', *Proc. 11th World Conf. Earthquake Engng.*, Acapulco, paper no. 1380, 1996.
10. M. Iiba, H. Mizuno, T. Goto and H. Kato, 'Shaking table test on seismic performance of confined masonry wall', *Proc. 11th World Conf. Earthquake Engng.*, Acapulco, paper no. 659, 1996.
11. K. Yoshimura, K. Kikuchi, Z. Okamoto and T. Sanchez, 'Effect of vertical and horizontal wall reinforcement on seismic behaviour of confined masonry walls', *Proc. 11th World Conf. Earthquake Engng.*, Acapulco, paper no. 191, 1996.
12. M. O. Moroni, M. Astroza and S. Tavonatti, 'Nonlinear models for shear failure in confined masonry walls', *The Masonry Soc. J.* (Boulder) **12** (2) 72–78 (1994).
13. M. Moroni, M. Astroza and P. Mesias, 'Displacement capacity and required story drift in confined masonry buildings', *Proc. 11th World Conf. Earthquake Engng.*, Acapulco, paper no. 1059, 1996.
14. M. Tomaževič and I. Klemenc, 'Seismic behaviour of confined masonry buildings. Part 2: shaking-table tests of model buildings M1 and M2—analysis of test results'. *Report ZAG/PI-95/06*, Ljubljana, 1996.
15. V. Turnšek and F. Čačovič, 'Some experimental results on the strength of brick masonry walls', *Proc. 2nd Int. Brick Masonry Conf.*, Stoke-on-Trent, 1970, pp. 149–156.
16. R. Žarnić and M. Tomaževič, 'Study of the Behaviour of Masonry Infilled Reinforced Concrete Frames Subjected to Seismic Loading', *Proc., 7th Int. Brick-Masonry Conf.*, Vol. 2, Melbourne, 1985, pp. 1315–1325.
17. R. Žarnić, 'Inelastic model of r/c frame with masonry infill—analytical approach', *Int. J. Engng Modelling* (Split) **7**(1–2), 47–54 (1994).
18. M. J. N. Priestley and D. O. Bridgeman, 'Seismic resistance of brick masonry walls', *Bull. the New Zealand Nat. Soc. for Earthquake Engng.* (Wellington) **7**(4), 167–187 (1974).